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Numerical analysis of foundation columns to support widening of embankments

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Abstract

Increased traffic volume has made it necessary to increase highway capacities by widening embankments and pavements. Adding a new embankment to an existing embankment induces additional stresses and deformations beneath the widened and existing portions of the embankment. Differential settlement may develop between and within the new and existing portions of the embankment, especially over soft soils. This differential settlement often causes pavement distress, such as longitudinal cracks or the drop-off (or sinking) of pavement sections. Different techniques have been adopted to remedy these problems, including the use of foundation columns, such as deep mixed columns, vibro-concrete columns, stone columns, and aggregate piers. However, design procedures for foundation columns constructed for this purpose are not well developed. The analyses of eight cases of column-supported widened embankments and two untreated foundations are presented in this paper. The factors considered include the consolidation of foundation soils under existing embankments and the spacing, region, and modulus of foundation columns. Two-dimensional finite difference software was used after the calibration of the model against a field case study and numerical analyses were conducted to investigate stresses and deformations of the widened embankments over soft soil with or without the remediation of foundation columns. The results presented in this paper include the vertical and the horizontal displacements, the maximum settlements, the transverse gradient change, and the distribution of the additional stresses induced by the widening. Recommendations are made for the design of foundation columns to remedy roadway pavement failure due to widening of embankments.
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1. Introduction

Increased traffic volume since construction has made it necessary to increase highway capacities by widening embankments and pavements. The 1989 Government's White Paper 'Roads for Prosperity' [1] indicated that "about 60% of the motorway network in England as well as some truck roads will need to be widened by the provision of additional lanes". Adding a new embankment adjacent to the existing embankment induces not only additional stresses and deformations under the widened portion but also those under the existing embankment. Differential settlement may develop between and within the new and existing embankments, especially over soft soils. This often causes pavement distress, such as longitudinal cracks or drop-off of pavement sections. Ling et al. [2] reported a number of roadway pavement distresses due to widening of existing roads in China. Fig. 1 shows typical pavement distresses after widening of roadways [2]. The collapse of the pavement section is typically induced by the instability of earth structures, such as retaining walls or slopes. Pavement cracking is often induced by excessive

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(a) Slope failure (collapse) **(b) Longitudinal crack on asphalt pavement**

(c) Longitudinal crack on concrete pavement **(d) Drop-off of pavement section**

Fig. 1. Pavement distresses after widening of roadways [2].

differential settlements. The drop-off (or sinking) of pavement sections commonly occurs when the differential settlement is excessive but the pavement structure is rigid and strong. Ling et al. [2] proposed design criteria for widened pavements by setting limits for the transverse gradient change of the embankment (Table 1). The transverse gradient change is defined as the percentage of the maximum differential settlement induced by widening divided by the span of the two positions on which the differential settlement is based. These design criteria were developed based on asphalt pavements over lime-flyash-aggregate base and lime-stabilized soil subbase and assuming tensile failure occurring at the bottom of the base course. The criteria consider the utilization of the existing pavement structures, the pavement joint condition, and the existing and new

Table 1
Design criteria for transverse gradient change of pavements after widening [2]

Utilization of existing pavement structure	Pavement joint condition	Limit of gradient change (%)	
		Existing embankment, g_e	Widened portion, g_w
No	NA	0.4	0.4
Yes	Separate	1.5	0.5
	Connected	1.5	0.5
		$-0.18 \leqslant g_w - g_e \leqslant 0.43$	

embankments. When the existing pavement is utilized, the transverse gradient change for the existing embankment is accumulated from both the construction and post-construction. For other conditions, however, the transverse gradient change is only induced during the post-construction of the pavement. A centrifuge test done by Wang and Huang [3] resulted in a transverse gradient change up to 4%, which is much higher than that provided in Table 1. The transverse gradient change is the same as the angular distortion used in foundation design. Marsh and Thoeny [4] suggested an angular threshold criterion of 0.3–0.5% for slab-on-grade structures considering expansive soil curling or settlement at a negligible to low damage level, which is close to that for widened embankments proposed by Ling et al. [2].

Different techniques (Fig. 2) have been used in practice to avoid, minimize or remedy distresses due to the widening of embankments, such as the use of lightweight backfill materials, geosynthetic reinforcement, over-excavation and replacement, installation of piles or foundation columns, preloading, and a combination of the above alternatives [5]. Foundation columns include but are not limited to deep mixed columns, vibro-concrete columns, stone columns, and aggregate piers, etc., which are the focus of this study. Forsman and Uotinen [6] investigated the effect of geosynthetic reinforcement on the settlements and horizon-

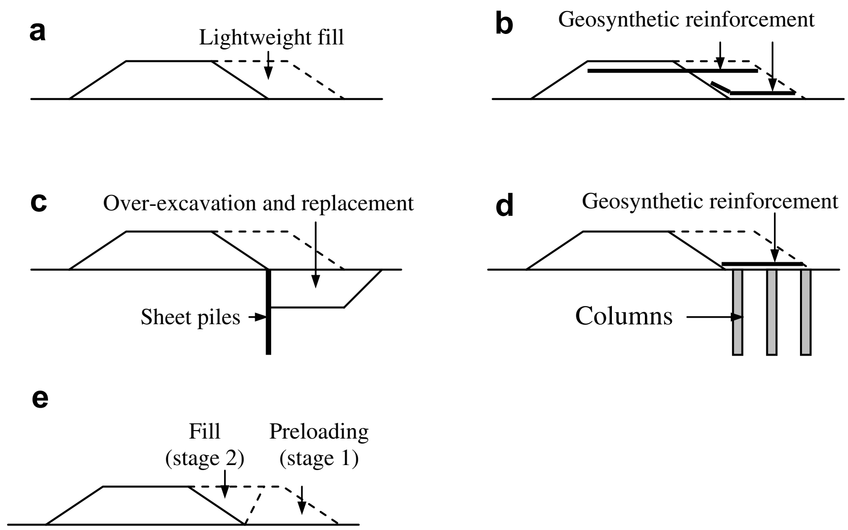

Fig. 2. Remedying techniques used in widening of roadways [5].

tal displacements of embankments after widening (Fig. 2b). Han and Akins [7] reported the use of vibro-concrete columns with geogrid layers above to support widened embankments in the US (Fig. 2d). Geosynthetic reinforcement may not be needed if the spacing of the columns is close and/or the height of the embankment is large. van Meurs et al. [8] presented a field study to investigate the effectiveness of the Gap-method for embankment widening (Fig. 2e). The basic principle of the Gap-method is to construct the widened portion in two stages. The new embankment fill is first placed with a gap to the existing embankment as surcharge for consolidation of soft soils in the first stage, which is also expected to provide lateral constraint to soils during the placement of the additional embankment fill in the second stage to fill the gap and unify the widened one to the existing one. However, this study concluded that the Gap-method did not provide any positive effect on the lateral movement of the embankment as compared with that in one-stage construction. This conclusion is slightly different from that obtained by Allersma et al. [9] using a small centrifuge study, in which a difference in deformations, especially horizontal displacements was observed between a regular horizontal widening method and the Gap-method.

Even though widening of embankments has been commonly adopted in practice, so far, very limited guidance for design is available for widening projects. Han et al. [5] proposed a simplified method for estimating the settlement of the widened embankment over a uniform foundation soil. This method cannot be applied for the situation where the soil under the existing embankment has better properties due to its consolidation or improved by ground improvement methods, such as foundation columns. So far, it is not well understood how foundation columns should be designed and laid out to reach desired performance. The objective of this study is to investigate the effects of installing foundation columns in the soil beneath the widened portion of the embankment and/or the consolidated soil under the existing embankment.

2. Calibration of numerical model

To ensure the reasonableness of the numerical model to be used for the parametric study, a field case study as described below was selected for the calibration of this numerical model. Due to the unavailability of a well-documented roadway widening case study to the authors' best knowledge and a search, a field study with a new embankment over geosynthetic and deep mixed columns [10] was adopted for this purpose. The details of this calibration can be found in the paper by Han et al. [11], therefore, a brief description of this calibration is presented below.

2.1. Brief project description

The selected project is a bridge embankment constructed on deep mixed (DM) columns beside the Sipoo River at Hertsby, Finland. Detailed description of this project can be found in the literature [10]. However, corrections and update on settlement measurements were made based on Forsman [12]. The soft foundation below the embankment consists of a 1–1.5 m thick crust, 10–14 m of soft clay, 0–6 m of silt, and 1–5 m of glacial till. The soft clay over the silt layer has an undrained shear strength of 10–15 kPa. The effective cohesion and friction angle are 8 kPa and 13°, respectively, determined from drained triaxial tests. The elastic moduli under drained and undrained conditions are 300–600 kPa and 3000–8000 kPa, respectively, also determined from the triaxial tests. The determined Poisson's ratio under drained conditions is 0.1–0.2. The embankment has a 50 mm thick asphalt layer, 200 mm thick crushed stone base course, 1050 mm gravel subbase, and 500 mm thick sand working platform above the existing ground (i.e., at the base of the embankment).

The soft foundation was improved with DM columns as shown in Fig. 3 to have enough bearing capacity and minimal compressibility. The columns have an average diameter of 800 mm. Cement and by-product based binder were used as an admixture and the admixture content was 130 kg/m^3. The top layer on the embankment shown in Fig. 3 is the pavement section (including asphalt, base course, and subbase course). The design shear strength of the columns is 150 kPa. One layer of woven geotextile and a 0.3 m sand layer were placed over the columns. The ultimate strength of this geotextile is 200 kN/m in both longitudinal and transverse directions. The secant stiffness of this geotextile layer is 1790 kN/m and 2120 kN/m at strains of 2% and 6%, respectively. The adjacent geotextile sheets were jointed together by seams. This constructed embankment was instrumented with horizontal hydrostatic profile gauges, settlement plates, and strain gauges on the geotextile sheet.

After 5 years, the measured maximum settlements were approximately 120 mm [12] as shown in Fig. 4. There was differential settlement between the DM column walls. The settlements had become relatively stable after two years since construction. The measured strains in the geotextile in the longitudinal and transverse directions of the embankment were 0–0.2% and 0–1%, respectively. The tensions in the geotextile corresponding to 0.2% and 1% strain are 3.6 kN/m and 18 kN/m, respectively.

2.2. Numerical modeling

A 2D finite difference method, incorporated in fast Lagrangian analysis continua (FLAC) Version 4.1 [13], was adopted in this study. The numerical model for calibration against this case study is presented in Fig. 5. Due to the symmetry of the problem, half of the section was used in the analysis. The DM columns with a wall pattern were modeled as two-dimensional soil–cement

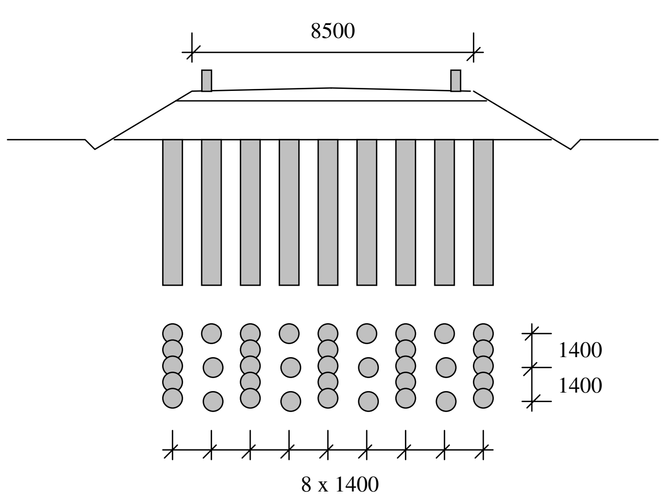

Fig. 3. Cross section of embankment and layout of DM columns [10].

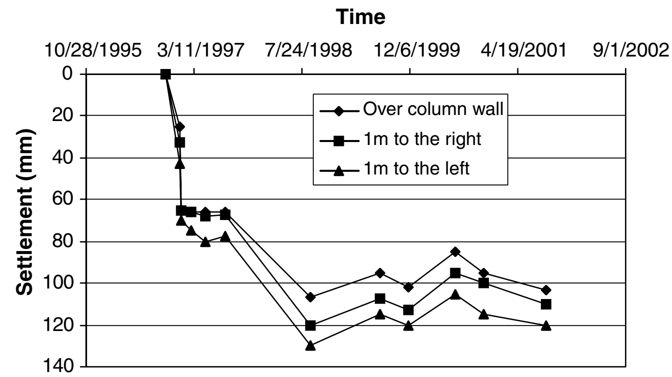

Fig. 4. Measured settlement with time [11].

walls. The wall at the centerline has only half-width. Two cases were investigated to evaluate the influence of the individual mid-columns between DM column walls: (1) without the mid-columns and (2) with the mid-columns. The case without mid-columns is the worst situation for design. In the case with mid-columns, the mid-columns are converted to an equivalent soil–cement wall as discussed later for the 2D analysis purpose. The actual behavior of the embankment over DM columns is expected to be between these two cases. The DM columns, the soft clay, the silt, and the embankment fill were modeled as elastic-perfectly plastic materials. The soil layers and the DM columns extend to the depth of the firm glacial soil. No deformation below the silt layer is assumed. A Mohr–Coulomb failure envelope was used as the failure criterion. A cable element was used to simulate the geotextile layer, which is located 0.3 m above the top of the DM columns. The properties of the soils, the DM columns, and the geosynthetic layer are provided in Table 2, most of them from Forsman et al. [10]. The elastic modulus of the DM columns was estimated based on the typical relationship of $E = 100\ q_u$ (e.g., Porbaha [14]; Bruce [15]), where q_u = unconfined compression strength (300 kPa). The effective thickness of the DM wall was estimated to be 0.7 m. The effective thickness was estimated based on the equivalency of the actual area of a series of DM columns to the area of the1 wall modeled in the analysis. The wall thickness for the mid-columns was taken to be the same but the equivalent elastic modulus and the equivalent cohesion of the mid-column walls were determined based on the area average of these parameters from the DM columns and the soft soil (i.e., $E_{eq} = E_c a_s + E_s(1 - a_s)$ and $c_{eq} = c_c a_s + c_s(1 - a_s)$, where E_{eq}, E_c, and E_s are the equivalent modulus and the moduli of the column and the soft soil, respectively; c_{eq}, c_c, and c_s are the equivalent cohesion and the cohesion of the column and the soft soil, respectively; a_s is the improvement ratio by the DM columns within the row of the individual columns). The crust near the ground surface was not considered in the numerical analyses since the crust might be disturbed during the

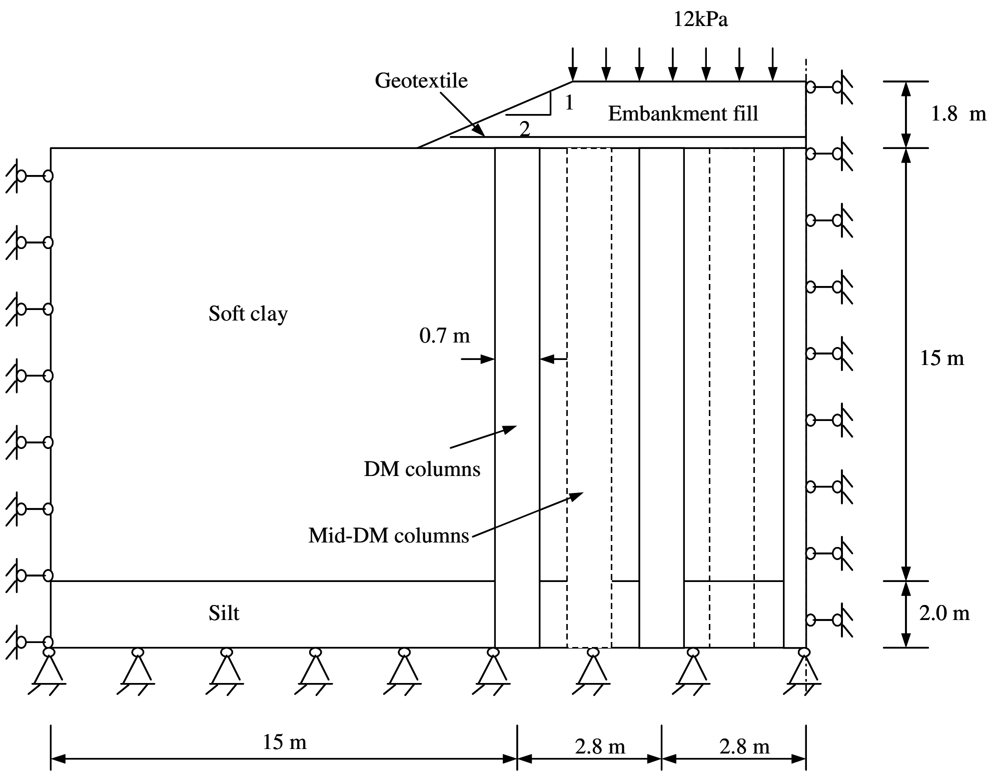

Fig. 5. Dimensions and boundary conditions in the numerical model.

Table 2
Material properties used in the numerical analysis

Material	E (MPa)	ν	γ (kN/m^3)	c' (kPa)	ϕ' (°)
Soft clay	0.3	0.2	14.8	8	13
Silt	1.6	0.33	20	5	20
Embankment fill	40	0.33	20	5	38
Platform fill (below geotextile)	20	0.33	20	5	32
DM column wall	30	0.3	20	150	0
Mid-DM column wall	15	0.3	20	79	0
Geotextile (cable)	$J = 1700$ kN/m, $c_i = 0.8$, $k = 85{,}000$ kN/m/m				

Note: E, elastic modulus; ν, Poisson's ratio; γ, unit weight; c', effective cohesion; ϕ', effective friction angle; J, tensile stiffness of geotextile; c_i, interaction coefficient between geotextile and sand; and k, interface shear stiffness between geotextile and sand.

installation of DM columns and this simplification is expected to yield conservative results.

The actual construction sequence was not reported in the reference (Forsman et al. [10]) and it was simulated by adding embankment fill in six layers with equal thickness (0.3 m). A surcharge of 12 kPa was used to simulate the traffic loading. The problem was analyzed based on drained conditions. The bottom boundary is fixed in both horizontal and vertical directions and the two side boundaries are fixed in the horizontal direction but free in the vertical direction.

2.3. Results and comparisons

Since the details of the numerical results for vertical displacements, tension in reinforcement, and vertical stresses were reported by Han et al. [11], herein are only the key results reported and compared. The maximum settlements and tension in the geotextile from the measurement and the numerical analyses are summarized in Table 3. As expected, the case without mid-columns over-predicts the maximum settlement between the DM column walls over the measured. The case with mid-columns yields a close agreement of maximum settlement between the DM columns to the measured. The calculated maximum settlement of the DM column wall is very close to the measured if the mid-columns are included.

As shown in Table 3, the numerical analyses yielded a maximum tension from 7.3 kN/m (with mid-column walls) to 18.3 kN/m (without mid-column walls), while the

Table 3
Comparisons of maximum settlement and tension in geotextile

	Measured	Numerical analysis	
		Without mid-columns	With mid-columns
Maximum settlement (mm)	110–120 (104)	196 (127)	110 (100)
Maximum tension (kN/m)	3.6–18.0	18.3	7.3

The number in the parenthesis ( ) represents the maximum settlement over the DM column wall.

measured maximum tension ranges from 3.6 kN/m to 18.0 kN/m. They are in a reasonably good agreement. It should be pointed out that the tension of 3.6 kN/m was measured in the longitudinal direction of the embankment rather than the transverse direction in the analyses. This comparison indicates that the calculated tension (18.3 kN/m) in the reinforcement in the transverse direction for the case without mid-columns is very close to the measured (18.0 kN/m) in the same direction. This agreement can be explained by the fact that the geotextile sheet between the gaps of the individual mid-DM columns actually has a long span between the column walls, which creates a similar situation to that without mid-columns modeled in the numerical analysis.

These comparisons prove that this numerical model is efficient and reasonable for the analysis of embankments over columns, therefore, it was adopted for the parametric study of foundation columns for remedying roadway failure due to widening of existing embankments. In this parametric study, geosynthetic reinforcement was not included and will be investigated in the future.

3. Parametric study

3.1. Numerical modeling

The cross-section, boundary conditions, and dimensions for the numerical model used for this parametric study are

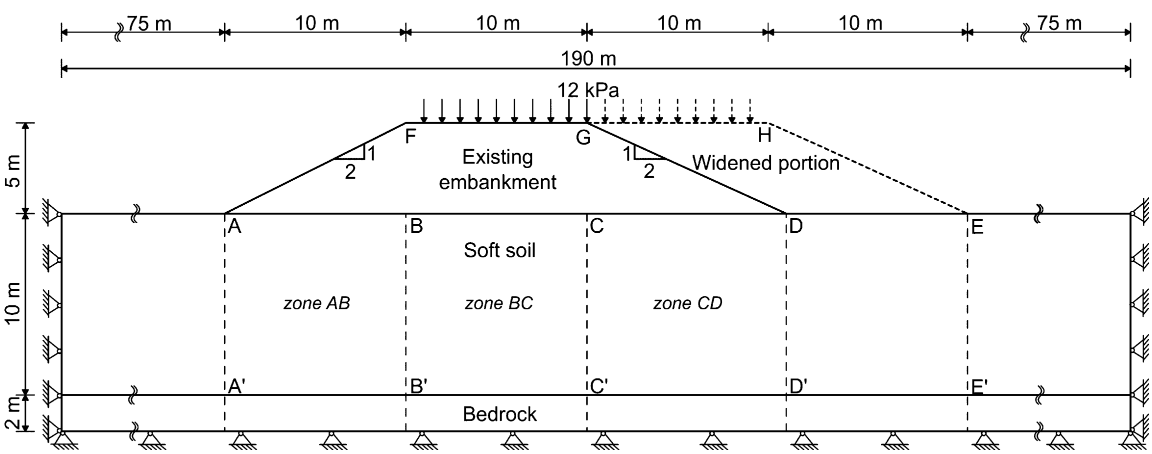

Fig. 6. Model for the numerical analysis of roadway widening.

Pattern	Case	Soil condition	Treatment	Column spacing, s (m)	Column modulus, E_p (MPa)	Soil and column properties
Uniform soft soil	1a	Uniform	No treatment	-	-	Embankment γ=20 kN/m^3 E=20 MPa ϕ=35°; c=0 ν=0.30
Uniform soft soil	1b	Uniform	Columns beneath only the widened portion	3.0	100	Soft soil *(Uniform)* γ=18 kN/m^3 E=5 MPa ϕ=25°; c=0 ν=0.30
Zone AB, Zone BC, Zone CD	2a	Non-uniform	No treatment	-	-	Soft soil *(Non-uniform ; zones AB & CD)* γ=18 kN/m^3 E=7.5 MPa ϕ=27.5°; c=0 ν=0.30
Zone AB, Zone BC, Zone CD	2b 2c 2d	Non-uniform	Columns beneath only the widened portion	3.0 2.0 3.0	100 100 200	Soft soil *(Non-uniform ; zone BC)* γ=18 kN/m^3 E=10.0 MPa ϕ=30°; c=0 ν=0.30
Zone AB, Zone BC, Zone CD	2e 2f 2g 2h	Non-uniform	Columns beneath both the slope of existing embank. and the widened portion	3.0 2.5 2.0-3.0 2.0	100 100 100 100	Bedrock E=1000 MPa ν=0.25 Columns γ=20 kN/m^3 E=100 MPa ϕ=45°; c=0 ν=0.25

Fig. 7. Parameters used in all the analyzed cases.

presented in Fig. 6. The columns were modeled as walls in the plane strain condition. The thickness of the column walls used in this analysis was 1.0 m. All the columns started from the base of the embankment and were embedded into the bedrock for 1 m. The cases, materials properties, soil conditions, and options of treatment are presented in Fig. 7. The analysis was conducted under a drained condition. For clarity, the existing embankment refers to ADGF, the widened portion refers to DEHG, and the widened embankment refers to AEHF. Embankment fill, soft soil, and columns are modeled as linearly elastic perfectly plastic materials with Mohr–Coulomb failure criteria. Bedrock is modeled as an elastic material with high modulus and the compression of the bedrock is negligible. The material properties were selected based on the typical reference values [16] and considering those used in the calibration case. There are two foundation soil conditions: uniform and non-uniform. The uniform soil condition is an ideal situation, which is the same as that investigated by Han et al. [5]. This condition also serves as a basis for investigating the effect of consolidated soil under the existing embankment, i.e., the non-uniform soil condition. The non-uniform soil condition includes three consolidated zones (AB, BC, and CD) and surrounding zones. The consolidated zones have better soil properties in terms of strength and modulus than the surrounding soil zones. The strength and modulus in Zone BC are higher than those in Zones AB and CD considering different overburden stresses under the center and the slope portions of the embankment. One of the primary purposes of this study is to demonstrate the magnitude of the effect of improved properties of consolidated soil under the existing embankment on the performance of the widened embankment. The options of treatment for the foundation soil include no treatment, columns beneath only the widened portion, and columns beneath both the connection side slope of the existing embankment and the widened portion. In this study, the columns underneath the existing embankment are installed prior to the construction of the widened portion. In realty, the columns in this zone may be installed before or during the construction of the widened section. The influence of the construction sequence of columns will be investigated in a future study. In addition, the effects of column spacing and modulus were examined in this study. This numerical analysis was conducted under a two-dimensional plane strain condition and the columns were treated as walls with a thickness of 1 m.

In the numerical modeling, the ground was first formed by applying a gravity force under a small strain condition. The small strain mode does not update the mesh coordinates. Any deformation induced during this process was zeroed out. Then the existing embankment was constructed and followed by a traffic loading of 12 kPa still under a small strain condition. The small strain mode was selected to ensure no distortion of mesh develops before the widening of the existing embankment. After the construction of the existing embankment and the application of the traffic loading, all the displacements were zeroed out again. This process simulated the completion of the consolidation of the existing embankment. If the consolidation of soil is considered in the analysis, the properties of Zones AB, BC, and CD are changed to the improved properties as shown in Fig. 7. If the foundation columns are considered, the corresponding zones are changed to the proper properties of columns. After this process, the widened portion and the traffic loading on the widened portion were added and analyzed in a large strain mode to simulate the widening of the embankment. The large strain mode updates the mesh coordinates due to displacements. The widened portion was placed in one stage and the analyses were conducted with the assumption of drained conditions. All the displacements induced during the last process resulted from the widening of the embankment.

3.2. Analysis of results

3.2.1. Vertical displacement contour and maximum settlement

Vertical displacement (settlement) contours for the embankment due to widening are presented in Fig. 8 for three cases. It is shown that the maximum settlement develops on the crest of the embankment due to the traffic loading, the compression of the embankment fill, and the slope movement in addition to the settlement at the base of the embankment. Fig. 8 shows that the differential settlements develop mainly in the widened portion and decrease from Case 2a (no column), Case 2d (columns under only the widened portion) to Case 2e (columns under both the connection side slope of the existing embankment and the widened portion). When the columns are constructed under only the widened portion, the location of the maximum displacement is shifted towards the existing embankment. The addition of columns under the connection side slope of the existing embankment (i.e., Zone CD in Fig. 6) controls and further minimizes the maximum displacement. In addition to the reduced displacements, the contours suggest a dramatic change in the displacement gradients, which will be further discussed in the section of "transverse gradient change".

The maximum settlements at the crest and the base of the widened embankments are presented in Table 4. The embankment fill settlement was calculated by subtracting the settlement at the base from that at the crest. It is shown that the fill settlement is close for all the cases and its average value is approximately 10 mm.

3.2.2. Settlement profile

The settlements at the base of the embankment induced by widening are presented in Fig. 9. Slight heave occurs below the left side slope of the existing embankment. The settlement profile for Case 1a is almost the same as that presented by Han et al. [5], in which elastic models were used for the all materials. This comparison implies that the embankment and the soft soil in Case 1a are mostly

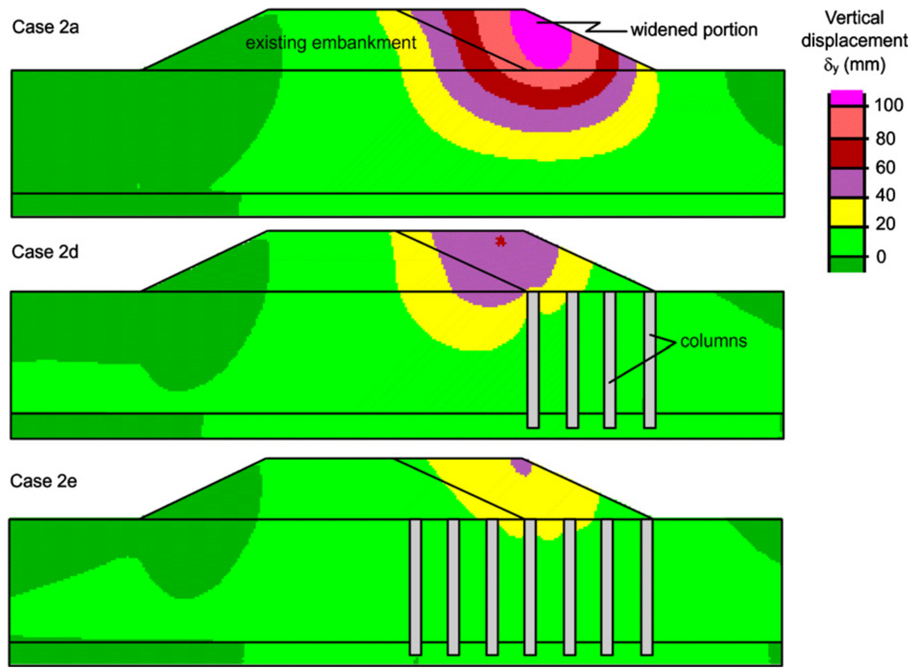

Fig. 8. Vertical displacement contours of widened embankments.

Table 4
Maximum settlements at the crest and the base of the widened embankments

Location	Case (unit: mm)									
	1a	1b	2a	2b	2c	2d	2e	2f	2g	2h
Crest	131	83	106	61	57	60	40	35	33	30
Base	120	72	99	50	47	48	30	25	21	19
Fill	11	11	7	11	10	12	10	10	12	11

in the elastic range. As Case 1b is compared with Case 1a (both under a uniform soft soil condition) in Fig. 9a, the use of columns not only reduces the maximum settlement but also pushes the location of the maximum settlement towards the existing embankment. In other words, the soft soil under the existing embankment plays an important role in the maximum settlement.

Fig. 9b shows the effect of the improved properties of the consolidated soil under the existing embankment. Case 2a with the consolidated soil has less settlement than Case 1a with the uniform soil.

Fig. 9c shows the effect of columns under only the widened portion for non-uniform soil conditions. Similar to Fig. 9a, the use of columns (Cases 2b and 2c) reduces the maximum settlement and shifts the location of the maximum settlement. It is also shown that the reduction of the column spacing from 3m (Case 2b) to 2m (Case 2c) only slightly reduces the maximum settlement. This phenomenon results from the fact that the maximum settlement is mostly contributed by the soft soil under the existing embankment. Another analysis for Case 2d was performed by changing the column modulus from 100 MPa to 200 MPa. No obvious difference is found due to the same reason which is related to the soft soil under the existing embankment. Therefore, the results of Case 2d were omitted in Fig. 9c.

Due to the influence of the soft soil under the existing embankment as discussed above, the columns installed under the connecting side slope of the existing embankment (i.e., Case 2e) reduce the maximum settlement of the widened embankment as shown in Fig. 9d. The location of the maximum settlement for Case 2e shifts back to the widened portion. This shift is because the soft soil under the widened portion has lower strength and modulus than that under the connecting side slope of the existing embankment. Comparing Case 2f with Case 2e indicates that further reduction of the column spacing from 3.0 m to 2.5 m reduces the maximum settlement as shown in Fig. 9e.

Case 2g in Fig. 9f includes columns with varying spacing from Section DD′ at 2.0 m towards Sections CC′ and EE′ at 3.0 m (the average spacing is 2.5 m). This varying spacing further slightly reduces the maximum settlement as compared with Case 2f. Case 2g had the least settlement among all the cases.

3.2.3. Horizontal displacement

Six representative cases for horizontal displacements are presented in Fig. 10. For uniform soft soil conditions, the soil left of Section DD′ (toe of the existing slope) moved left away from the widened portion and the soil right of DD′ moved right away from the existing embankment as shown in Fig. 10a. This result is in good agreement with that found by Han et al. [5]. The horizontal displacements along the DD′ section is nearly zero. The largest horizontal displacements (towards the left toe of the existing embank-

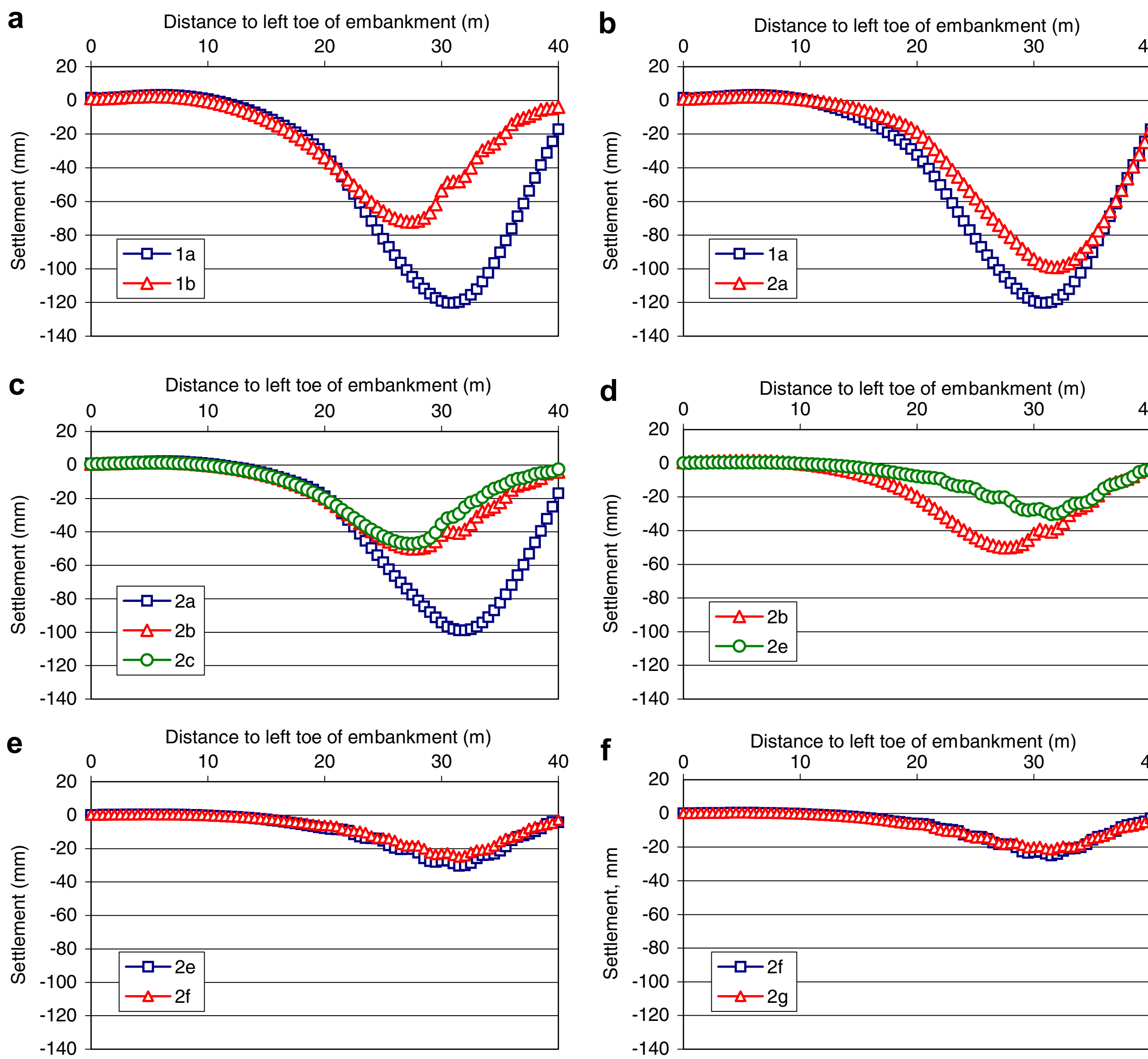

Fig. 9. Settlement profile at the base of the widened embankment.

ment) occur at the CC′ section. The magnitude of the horizontal displacements decreases from the CC′ section to the BB′ and the AA′ sections. Since the lower layer is bedrock, minimal horizontal displacements develop in this layer.

As compared with Case 1a in Fig. 10a, Fig. 10b shows that the installation of columns under the widened portion reduces the horizontal displacements at the EE′ section by providing confinement but changes the direction and increases the horizontal displacements at the DD′ section. The change of the direction in the horizontal displacements results from the rotation of the columns under only the widened portion due to their rigidity under the weight of the widened portion. Fig. 10b also shows that the columns have little effect on the horizontal displacements of the soft soil under the existing embankment.

Fig. 10c shows that the improved properties of the consolidated soil under the existing embankment reduce the horizontal displacements under the existing embankment. The horizontal displacements at the DD′ section moving towards the widened portion result from the fact that the soft soil under the widened portion is weaker than the consolidated soil. Fig. 10c also shows that the improved properties of the consolidated soil have little effect on the horizontal displacements at the toe of the widened portion (i.e., the EE′ section).

As compared with Case 2a, Fig. 10d shows that the installation of columns reduces the horizontal displacements under the widened portion.

As compared with Case 2b, further installation of columns under the existing embankment reduces not only the horizontal displacements under the existing embankment but also those under the widened portion.

Case 2g includes columns with varying spacing from Section DD′ at 2.0 m towards Sections CC′ and EE′ at 3.0 m. This column configuration resulted in the lowest horizontal displacements among all cases evaluated.

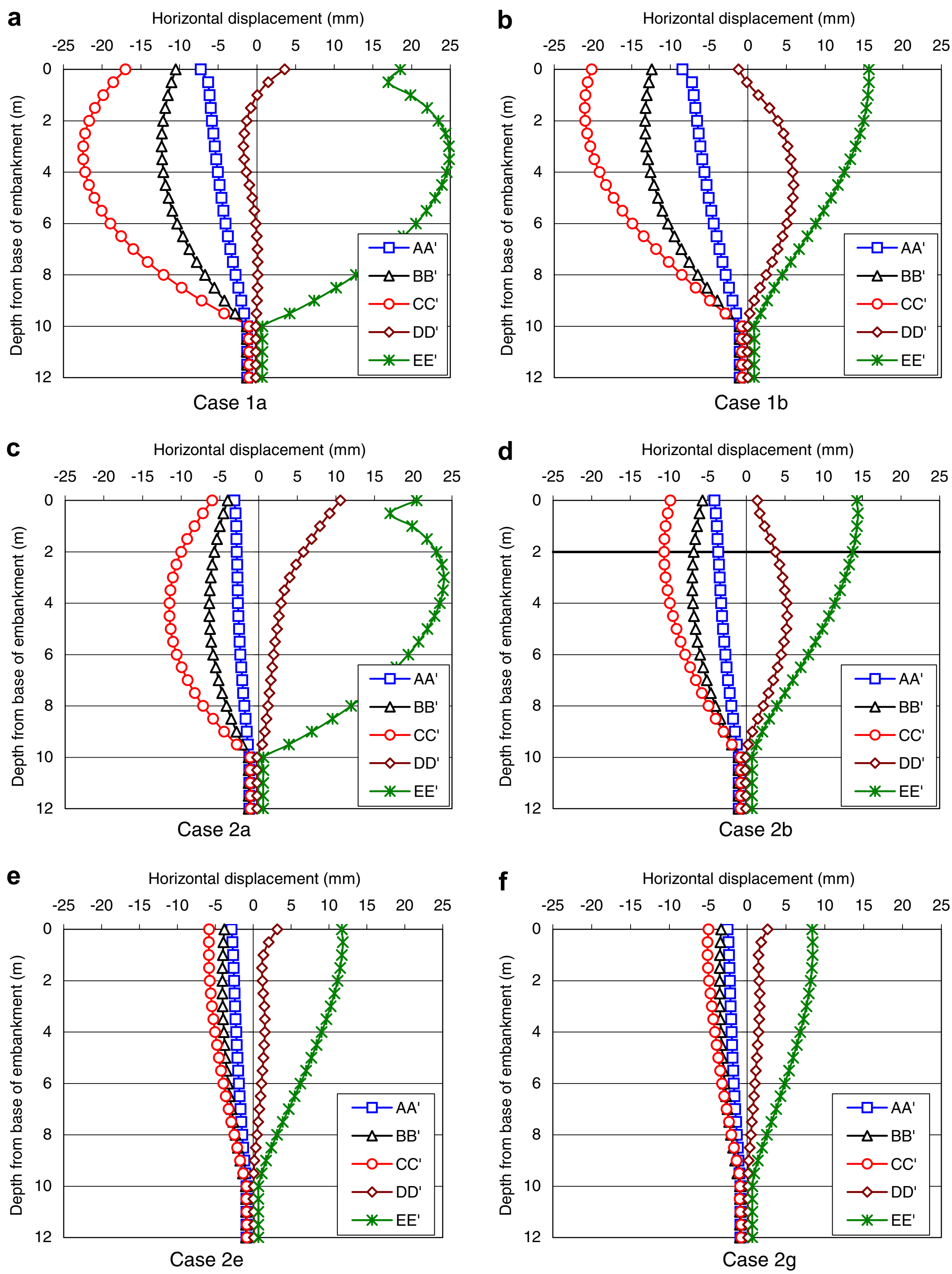

Fig. 10. Horizontal displacement below the widened embankment.

3.2.4. Transverse gradient change

The transverse gradient change is defined as the distortion (i.e., differential settlement/distance) of the pavement in percent due to widening of the embankment. This distortion is also the slope change of the pavement in the transverse direction. In this numerical analysis, the pavement of the existing embankment is assumed to be horizontal initially; therefore, the slope change of the pavement from the initial position is equal to the current slope after widening. Fig. 11 presents the settlement profiles of six cases at the crest of the embankments. The transverse gradient change for the existing embankment and the widened portion can be determined as the slopes of the two straight lines as shown in Fig. 11. The results of the transverse gradient changes are summarized in Table 5. Fig. 11 and Table 5 both show the transverse gradient change for the widened portion is always greater than that for the existing embankment. It is shown that the improved properties of the consolidated soil (Case 2a) are helpful for minimizing the transverse gradient change for the existing embankment and the widened portion. The installation of columns reduces these gradient changes as well. Installation of columns beneath the widened portion (Cases 2b, 2c, and 2d) resulted in substantial reduction (≈25–35%) in gradient change. Reduction was even greater (≈60–80%) when columns were installed beneath the widened portion and the preexisting embankment slope (Cases 2e, 2f, 2g, and 2h). Based on the recommendation of Ling et al. [2], if the limit of 0.4% is used herein, only the cases with columns under both the connecting side slope of the existing embankment and the widened portion (i.e., Cases 2e, 2f, 2g, and 2h) meet this criterion. These cases also meet the requirement of $-0.18 \leqslant g_w - g_e \leqslant 0.43$ (see Table 1), where g_w and g_e are the transverse gradient changes for the widened portion and the existing embankment, respectively.

3.2.5. Vertical stress

The vertical stress distributions at the depth of 0.25 m below the base of the widened embankment are presented in Fig. 12. Fig. 12 shows that the widening of the existing embankment slightly disturbs the vertical stress distribution for the cases without columns (Cases 1a and 2a), i.e., the vertical stress changes at the interface between the existing and widened portions. The solid lines in Fig. 12a

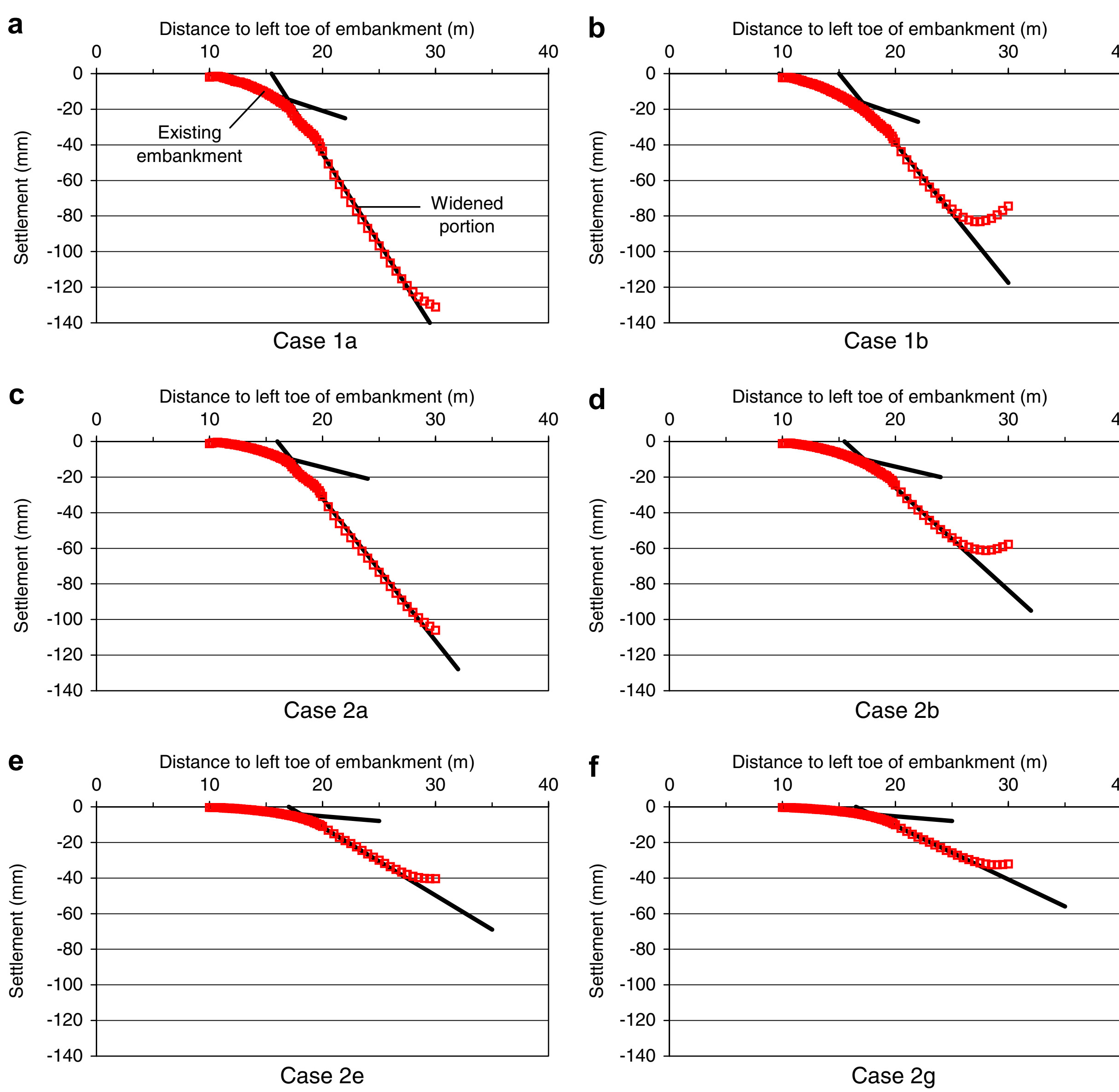

Fig. 11. Transverse gradient change.

Table 5
Transverse gradient change (%)

	Case									
	1a	1b	2a	2b	2c	2d	2e	2f	2g	2h
Existing, g_e	0.21	0.22	0.16	0.15	0.15	0.15	0.05	0.05	0.05	0.04
Widened, g_w	1.00	0.78	0.80	0.58	0.63	0.57	0.38	0.33	0.30	0.28
$g_w - g_e$	0.79	0.56	0.64	0.44	0.49	0.42	0.32	0.28	0.26	0.24

represent the approximation of vertical stresses based on the overburden stress ($\gamma\ H_x$, γ = the unit weight of the embankment fill and H_x = the height of the embankment fill at each location, x, from a depth of 0.25 m below the base of the embankment) of the embankment and the traffic surcharge (q = 12 kPa for this analysis) on the crest. The approximation of the vertical stress distribution agrees reasonably well with the numerical results. The spikes in these figures indicate the stress concentration on the columns. Fig. 12b shows that the improved properties of the consolidated soil do not change the vertical stress distribution. Even though stress concentrations exist on the columns, the average vertical stress for these cases is still close to those without columns, as expected. The analysis shows that the vertical stresses on the columns are less than their strengths, therefore, columns remain elastic under the widened embankment loads.

3.2.6. Shear stress

The shear stress distribution at the base of the widened embankment is presented in Fig. 13. The positive shear stress acts towards the right direction (i.e., the widened portion direction) while the negative shear stress acts towards the left direction (i.e., the existing embankment direction). However, the shear resistance provided by the soft soil or columns is opposite to these directions. As shown in Fig. 13a for Case 1a, the widened embankment does not have a symmetric shear stress distribution as the existing embankment as discussed by Han et al. [5]. Rather, the widening of the embankment induces more shear stress under the widened portion. It is reasonable that the induced shear stress under the widened portion acts towards the rightward direction. The comparison of the results for Cases 1a and 2a shows that the improved properties under the existing embankment have little effect on the shear stress distributions. For the cases with columns, high shear stresses develop around the column locations but low shear stresses remain in the soft soil between the columns. In addition, in front of the columns for Cases 1b, 2b, and 2c (i.e., at the distance from 23 m to 30 m to the left toe of the exiting embankment), there is a high shear stress zone due to the resistance of the columns.

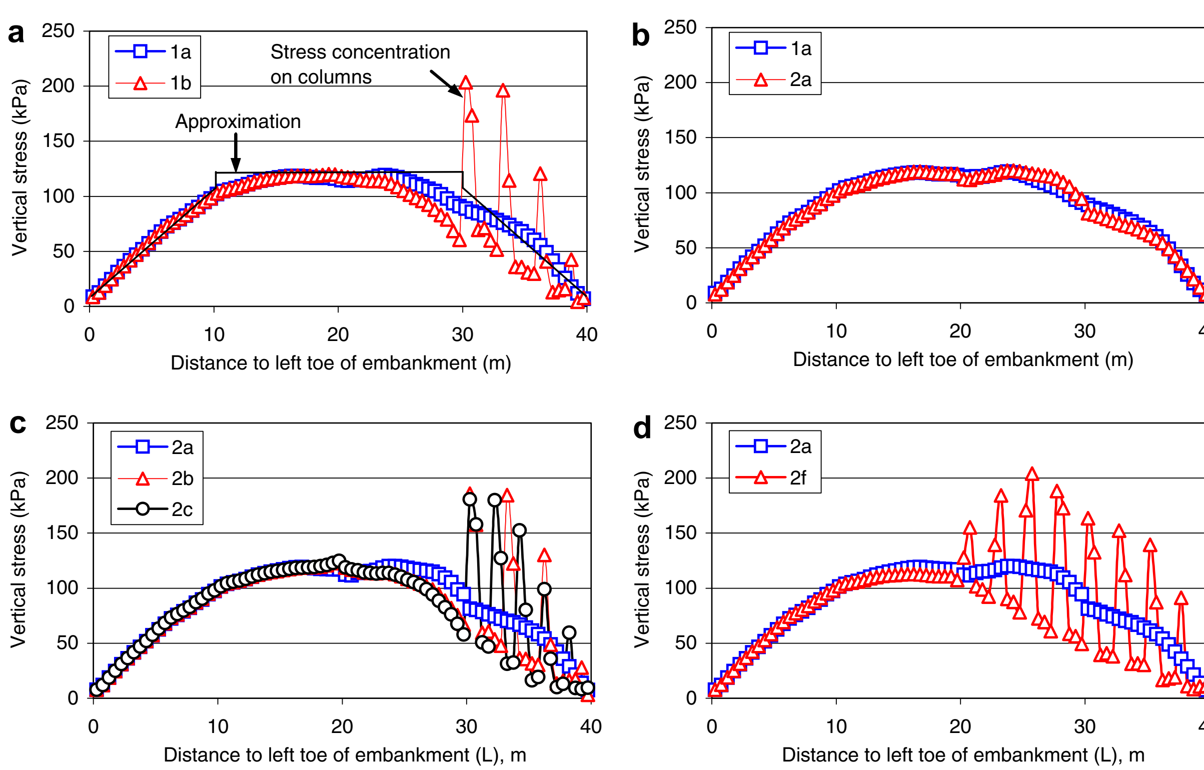

Fig. 12. Vertical stresses at the base of the widened embankment.

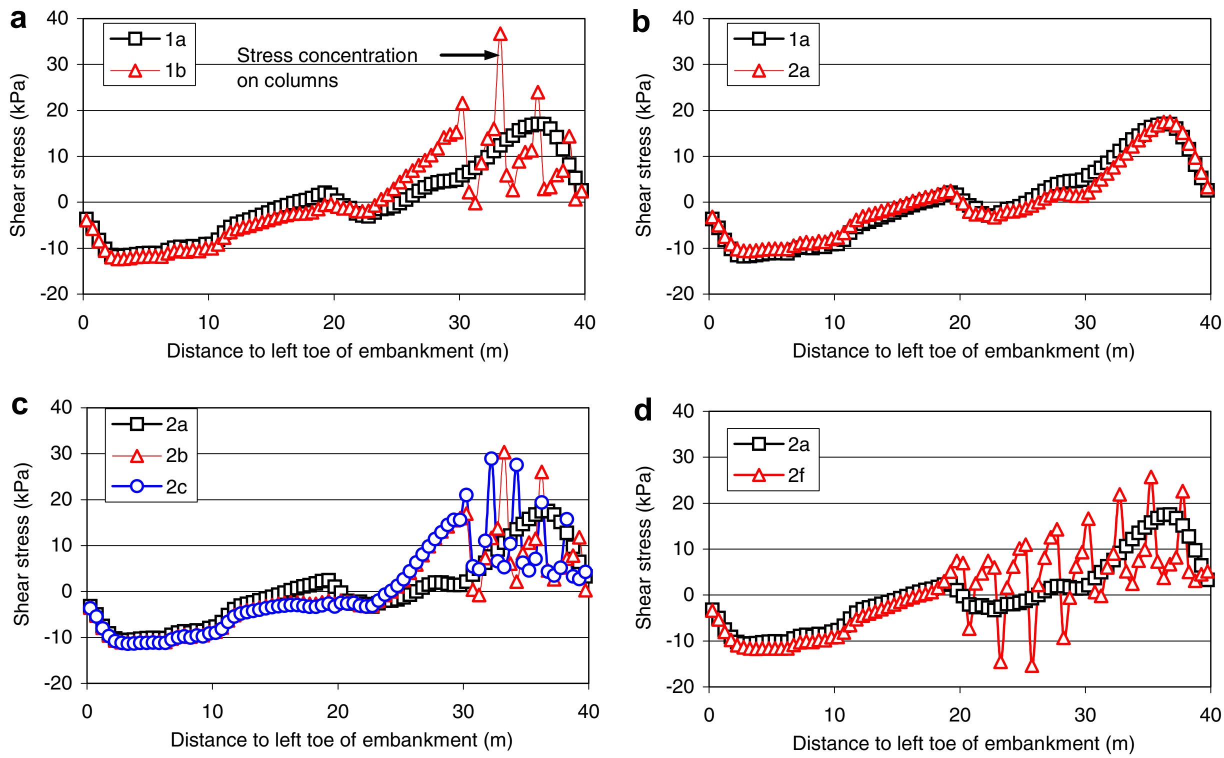

Fig. 13. Shear stresses at the base of the widened embankment.

These phenomena imply that columns can provide substantial shear resistance to the lateral force induced by widening of the existing embankment. Case 2f in Fig. 13d shows that the columns under the connecting side slope provide shear resistance towards the right direction while the columns under the widened portion provide shear resistance towards the left direction.

4. Conclusions

Widening of existing embankments over soft soil presents important geotechnical and pavement problems which have not been well addressed previously. Field observations have shown that cracks, drop-off of pavements, and collapse of embankment slopes often occur after widening of embankments. This paper presents the results of a numerical study of the stresses and deformations generated from widening embankments after the calibration of the numerical model. This study shows that consolidated soil with improved properties under the existing embankment reduces the maximum settlement, the horizontal displacement, and the transverse gradient change but has little influence on the vertical and shear stress distributions. These reductions can also be achieved by installing columns under the widened portion and/or the connecting side slope of the existing embankment. In the case of column installation under only the widened portion, the maximum displacements shift to the connecting side slope of the existing embankment. Further installation of columns under the slope of the existing embankment controls these displacements. The best performance can be obtained by optimizing the column spacing under the connection side slope of both the existing embankment and the widened portion. In this study, the transverse gradient change for the widened portion is always greater than that for the existing embankment. The transverse gradient change should be controlled to avoid possible failure or damage of roadways after widening. This study also shows the vertical stresses at the base of the widened embankment can be approximated by summing the overburden stresses created by the embankment and the surcharge on the crest of the embankment. Widening the embankment increases shear stresses in the existing embankment towards the widened portion. Foundation columns can provide shear resistance to the shear stress induced by widening of the embankments.

References

[1] The Highways Agency, the Scottish Office Development Department, the Welsh Office, the Department of the Environment for Northern Ireland. Geotechnical Considerations and Techniques for Widening Highway Earthworks; 1991.

[2] Ling JM, Qian JS, Huang QL. New technologies of subgrade widening for highways. Internal Report, Tongji University and Changsha Science and Technology University, 2003. [in Chinese]

[3] Wang H, Huang X. Centrifuge model test and numerical analysis of embankment widening on soft ground. In: Proceedings of the 8th international conference on applications of advanced technologies in transportation engineering, Beijing, China; 2004. p. 549–53.

[4] Marsh ET, Thoeny SA. Damage and distortion criteria for residential slab-on-grade structures. J Perform Constr Facilit 1999;13(3):121–7.

[5] Han J, Huang J, Liu SY, Hong ZS. Stresses and deformations induced by widening of existing embankments over soft soils. In: Proceedings of the international symposium of lowland technology, Saga, Japan; September 14–16, 2006. p. 201–5.
[6] Forsman J, Uotinen VM. Synthetic reinforcement in the widening of a road embankment on soft ground. In: Barends et al., editor. Proceedings of the geotechnical engineering for transportation infrastructure; 1999. p. 1489–96.
[7] Han J, Akins K. Case studies of geogrid-reinforced and pile-supported earth structures on weak foundation soils. In: O'Neill MW, Towsend FC, editors. Proceedings of the international deep foundation congress. Geotechnical Special Publication No. 116 – Deep Foundations 2002. Orlando (USA): ASCE; 2002. p. 668–79.
[8] van Meurs ANG, van den Berg A, Venmans AAM, Zwabenburg C. Embankment widening with the Gap-method. In: Barends et al., editor. Proceedings of the geotechnical engineering for transportation infrastructure; 1999. p. 1133–8.
[9] Allersma HGB, Ravenswaay L, Vos E. Investigation of road wideing on soft soils using a small centrifuge. J Transport Res Rec 1994;1462:47–53.
[10] Forsman J, Honkala A, Smura M. Hertsby case: a column stabilised and geotextile reinforced road embankment on soft subsoil. In: Bredenberg H, Holm G, Broms BB, editors. Dry mix method for deep soil stabilization. Rotterdam: Balkema; 1999. p. 263–8.
[11] Han, J, Huang, J, Porbaha, A. 2D numerical modeling of a constructed geosynthetic-reinforced embankment over deep mixed columns. ASCE Geotechnical Special Publication (GSP) No. 131: Contemporary Issues in Foundation Engineering, ASCE GeoFrontiers, Austin (TX); Jan. 24–26, 2005.
[12] Forsman J. Geovahvistetutkimus – Koerakenteiden loppuraportti 1996–2001, Tiehallinto, Helsinki; 2001. 122p.
[13] Itasca Consulting Group, Inc. FLAC/Slope user's guide, 1st ed.; 2002. 82p.
[14] Porbaha A, Shibuya S, Kishida T. State of the art in deep mixing technology: part III Geomaterial characterization. Ground Improv 2000;3:91–110.
[15] Bruce DA. An introduction to the deep mixing methods as used in geotechnical applications – vol. III: The verification and properties of treated ground. FHWA-RD-99-167; 2001. p. 455.
[16] Budhu M. Soil mechanics & foundations. Wiley; 2000. 586p.